Hutsonville Bridge
Spanning the Wabash River on State Route 154
in Sullivan County, Indiana, (to the village
of Hutsonville, in Crawford County, ~~Indiana~~ Illinois)

HAER No. IN-59

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PHOTOGRAPHS

WRITTEN HISTORICAL AND DESCRIPTIVE DATA

Historic American Engineering Record
Mid-Atlantic Regional Office
National Park Service
U. S. Department of the Interior
Philadelphia, Pennsylvania 19106

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HISTORIC AMERICAN ENGINEERING RECORD

Hutsonville Bridge

HAER No. IN-59

Location:	Spanning the Wabash River on State Route 154 in Sullivan County, Indiana, to the village of Hutsonville, in Crawford County, Illinois UTM: 16.443450 4329010 (East) 16 443330 4328970 (West) Quad: Hutsonville, Indiana Section 29 T8N, R11W
Date of Construction:	Construction began on January 27, 1939; bridge was dedicated on November 18, 1939; completed December 1939
Builder/Designer:	Designer: David B. Steinman Engineer: R. V. Milbank Engineering Firm: Robinson and Steinman, New York Constructed by: Wisconsin Bridge & Iron Company Milwaukee, Wisconsin Steel Fabricator: Vincennes Steel Corporation Vincennes, Indiana Subcontractor: Charles J. Glasgow Detroit, Michigan
Present Owner:	Indiana Department of Highways Indianapolis, Indiana 46204
Present Use:	Vehicular bridge
Significance:	This self-anchoring bridge is one of the few of its kind ever constructed in the United States. The bridge was determined eligible for inclusion on the National Register of Historic Places on July 16, 1987.
Project Information:	This documentation was undertaken in June 1988 to be in compliance with the Memorandum of Agreement among the Federal Highway Administration, the Advisory Council on Historic Preservation, and the Indiana State Historic Preservation Officer as a mitigative measure prior to the demolition of the bridge.

Linville R. Sadler, Chief
Division of Location and Environment
Indiana Department of Highways
Indianapolis, Indiana

The Hutsonville Bridge was designed by David B. Steinman, a principal in the engineering consulting firm of Robinson and Steinman. The firm, located at 117 Liberty Street in New York City, New York,[28] was prominent in the early 20th century, specializing in the design of suspension bridges.[30] Steinman had been a student of J. S. Roebling, the man who designed and supervised the construction of the Brooklyn Bridge.[29] Steinman went on to become one of the country's preeminent bridge engineers from the 1920s to the late 1950s.[30] He died on August 21, 1960.[4] Some of the more famous bridges that Steinman is given credit for designing include:

1. Henry Hudson Bridge in New York.
2. Carquinez Strait Bridge near San Francisco.
3. MacKinac Strait Bridge in upper Michigan.
4. St. John's Bridge in Oregon.
5/ Mt. Hope Bridge over Naragansette Bay in Rhode Island.[30]

The suspension bridge at Hutsonville is of an unusual design. It is unique in character by the fact that its suspension spans are self-anchored by virtue of the connection of the cables to the longitudinal stiffening girders rather than to concrete anchorage foundations.[26] From there comes the term "self-anchoring suspension Bridge." The Hutsonville Bridge was only the fifth of its type to be constructed in the United States.[5 26] The others built prior to the Hutsonville Bridge are three in Pittsburgh, Pennsylvania, and one over the Niangua River in Missouri.[1 31] Also, this bridge is the last vehicular suspension by bridge of any type on Indiana's highway system.[29 32]

The contract for the construction of the bridge was let on November 25, 1938.[16] It was built by a revenue bond issue, carrying 4-1/2 percent interest and a maturation date of November 1, 1958.[7] The bond payments totalled $300,000 and were sold for 90 cents on the dollar.[7 9 31] The bridge was completed in December 1939 at a final cost of $256,798.16.[9] It was constructed pursuant to Chapter 114, the Acts of 1929 and operated thereunder until 1937, at which time operation was continued pursuant to Chapter 141 of the Acts of 1937.[21 31] The bridge was operated until July 1, 1960, by the Sullivan County (Indiana) Bridge Commission, which was established by an act of the Indiana legislature in 1929.[1] Authority for the construction of this bridge was created in 1929 by the citizens of Sullivan County, Indiana, with the appointment of a three-man bridge commission.[1] This commission was empowered to hire engineers, sell bonds and perform all of the required business transactions.[1] The bridge was to be operated as a toll bridge, in order to pay off the bonds which were issued by the trust indenture dated November 1, 1938, with the National City Bank of Evansville (Indiana).[7]

In 1930, engineers were hired to design a bridge for a crossing at Merom, located about six miles southeast of Hutsonville.[1] The river bank on the Indiana side of the Wabash was high and satisfactory.[1] However the approach from Illinois was across several miles of frequently flooded, very low ground.[1] The Sullivan county Bridge Commission requested that the Illinois Highway Commission assist this project by constructing an all-weather approach road.[1] Illinois declined due to excessive costs.[1] This site was abandoned.[1] Eight years later (no reason available for length of time), a suitable crossing site with approaches that remained above flood waters was chosen at Hutsonville.[1] The west bank was naturally above flood water elevation, and the east approach was protected by a high levee.[1] The plans were revised for this particular crossing site and construction was soon to begin. The Secretary of War and the Chief of Engineers of the United States approved the location, plans and specifications of the bridge.[7 31]

The bridge offered a shorter route for several Indiana towns and cities to St. Louis, Missouri, as well as providing another outlet for the local coal field.[1]

The groundbreaking ceremony for the bridge was held on January 27, 1939.26 The bridge required a 242-day construction period.[26] The general contractor was the Wisconsin Bridge and Iron Company of Milwaukee, Wisconsin.[6 26 31] Charles J. Glasgow of Detroit, Michigan, was the subcontractor responsible for the construction of the floor and other miscellaneous work.[6] Steelwork was fabricated by the Vincennes Steel Corporation of Vincennes, Indiana.[6] The resident engineer was R. V. Milbank.[6 26 31]

The bridge was built to connect Illinois Road No. 1 and Crawford County Road No. 63 with Sullivan County Road No. 41 and Indiana Road No. 154.[26] It was dedicated on November 18, 1939, with between 5,000 and 10,000 people attending the ceremonies.[22 25] This was and still is the only bridge over the Wabash River for approximately 60 miles between Terre Haute and Vincennes.[22 23] It was not opened for traffic for at least 10 days after its dedication because Indiana State Road 154 was not yet completed.[22 24] The bridge replaced a ferry that was in operation a few hundred feet downstream to the south.[27]

In 1956, difficulty in paying off the debt of the bridge on time by the maturity date of the bonds was anticipated and eventually encountered. An agreement between the Sullivan County Bridge Commission and the bondholders had to be reached.[16] They were asked to waive their rights of foreclosure and allow the bridge to operate and collect tolls after the maturity date, so that the bonds could be successfully retired.[16]

Years earlier, in 1946, attempts were made by the Sullivan County Bridge Commission to have the Indiana State Highway Commission purchase the bridge and pay the bondholders off several years early. Ralph F. Gates, Governor of Indiana, even requested that this be investigated.[8] However, the Deputy Attorney General of Indiana was of the opinion that, since the construction of such interstate bridges involved the authority of both the Federal Government and the governments of the states affected, the powers granted to the Indiana legislature to construct highways could not logically be construed as conferring power on the Indiana State Highway Commission to purchase such bridges.[9 16]

By July 1, 1960, all of the bonds issued to raise revenue for the construction of the bridge and the interest thereon were paid and retired.[17 18] The bridge was then considered free and was accepted by the State of Indiana, as provided by the previously-mentioned Acts of 1929 and 1937, as part of the Indiana State highway system.[19] At this time, the Indiana State Highway Commission assumed the responsibility to perform the required maintenance and upkeep on the bridge.[19] A short time after this, a joint maintenance agreement with the State of Illinois was entered into, so that the costs of maintenance and repair could be equitably divided.[19 20]

One item of interest is that sometime in mid-1953 a controversy arose concerning the payment for engineering services (periodic inspections) to the State Highway Commission of Indiana by the Sullivan County Bridge Commission.[12] On August 8, 1950, a contract, agreeing to the compensatory payments for engineering services by a member or members of the State Highway Commission of Indiana, was signed by the chairman of the Sullivan County Bridge Commission, the chairman of the States Highway Commission of Indiana, and the Vincennes District Engineer.[11] The contract essentially stated that such engineering services shall be paid for as long as such an arrangement was mutually agreeable to both parties.[11] In letters dated July 13 and 18, 1953, the Secretary-Treasurer of the Sullivan County Bridge Commission, obviously for some reason unaware of the aforementioned contract, was mildly outraged at being charged for these engineering services for a bridge that would soon become the property of the State of Indiana.[12 13] He thereupon proceeded with a threat to charge every state-owned vehicle and piece of equipment which crossed the bridge, including state police vehicles.[12 13] This problem apparently was resolved. On December 23, 1953, a mutual agreement between the State Highway Department of Indiana and the Sullivan County Bridge Commission was reached.[14] Apparently, sometime after May 18, 1954, the agreement was signed by the chairman of the State Highway Department of Indiana.[15]

Detailed information on the specifications and dimensions of the Hutsonville Bridge can be seen on the plan sheets (see HAER photographs number IN-59-49 to IN-59-75). The bridge can be generally described as a self-anchoring suspension bridge of three spans (150 by 350 by 150 feet), with a 20-foot floor and 2-foot sidewalks on either side.[6] The towers are 70 feet above the main piers and 86 feet above the mean low water level.[23] The viaduct to the bridge is 352 feet long, making the total length of the structure 1,002 feet.[6] The preference of a self-anchored type of suspension bridge over that of an externally anchored bridge was dictated by the varying depth of sand on the Indiana side, which was up to 75 feet.[6] A brief explanation of how a self-anchoring suspension bridge works follows the attached endnotes. Also attached are items 1 and 6 listed in the endnotes. They provide more specific detail concerning the construction of this bridge.

Demolition of this historic bridge is scheduled to take place within 30 days of the opening of the replacement structure. At this time, it appears that the demolition will occur sometime in late August 1988.

ENDNOTES

1 Orvills W. Eusey, The Construction of a Self Anchoring Suspension Bridge, thesis submitted to Purdue University, June 1940, 32 pages.

2 Determination of Eligibility Notification, document from the Keeper of the National Register of Historic Places, 1 page.

3 D. B. Steinman and S. R. Watson, Bridges and Their Buildsrs, revised edition (original edition 1941), (Dover Publications, Inc., New York), p. 335.

4 Plowden, David, Bridges: the Spans of North America, (W. W. Norton, New York, 1974), PP. 290-291.

5 "Hutsonville Bridge," Engineering News-Record, Vol. 124, No. 7, February 15, 1940, p. 266.

6 C. H. Gronquist, "Self-Anchored I-Girder Suspension Bridge," Engineering News-Record, Vol. 124, No. 25, June 20, 1940, pp. 849-851.

7 Trust Indenture (document) between the Sullivan County Bridge Commission and the National City Bank of Evansville (Indiana), November 1, 1938.

8 Letter from Ralph E. Gates, Governor of Indiana to John H. Lauer, Chairman, Indiana State Highway Commission (ISHC), April 22, 1946.

9 Letter from J. R. Cooper, Engineer of Bridges (ISHC) to C. E. Vogelgesang, Chief Engineer (ISHC), November 6, 1946.

10 Letter from Charles L. Davis, Jr., Secretary-Treasurer of the Sullivan County Bridgs Commission (SCBC) to Lloyd E. Poindexter, Vincennes District Engineer, July 18, 1953.

11 Contract for Engineering Work, document between the Sullivan County Bridge Commission and the ISHC, August 8, 1950.

12 Letter from C. L. Davis, Jr. to Lloyd E. Poindexter, July 13, 1953.

13 Letter from C. L. Davis, Jr. to Lloyd E. Poindexter, July 18, 1953.

14 Letter from C. E. Vogelgesang, to D. E. Walker, Secretary, ISHC, January 6, 1954.

15 Memorandum from J. R. Cooper, Engineer of Bridges, ISHC, to C. E. Vogelgesang, May 18, 1954.

16 Letter from J. R. Cooper to Virgil W. Smith, Chairman, ISHC, June 29, 1956.

17 Letter from J. R. Cooper, Assistant Chief Engineer, ISHC, to John Peters, Chairman, ISHC, August 25, 1960.

18 Letter from Dana Pigg, SCBC, to John Peters, June 27, 1960.

19 Letter from M. A. Newlin, Office Engineer of Maintenance, ISHC, and Joseph E. Wilson, Superintendent of Maintenance, ISHC, to George J. Chamberlain, Vincennes District Engineer, ISHC, June 28, 1960.

20 Letter from R. R. Bartelsmeyer, Chief Highway Engineer (Illinois) to C. E. Vogelgesang, Chief Engineer, ISHC, august 22, 1960.

21 Letter from J. R. Cooper to Edwin K. Steers, Attorney General (Indiana), August 30, 1960.

22 "Hutsonville Bridge," The Indianapolis Sunday Star, November 19, 1939, 10.

23 "Hutsonville Bridge," Sullivan Daily Times, September 11, 1939, 1.

24 "Hutsonville Bridge," Sullivan Daily Times, November 14, 1939, 1.

25 "Hutsonville Bridge," Sullivan Daily Times, November 22, 1939, 1.

26 "Hutsonville Bridge," The Sullivan Union, November 9, 1939.

27 Hutsonville Bridge," The Sullivan Union, November 23, 1939.

28 Specifications, Proposal and Contract, document between the Sullivan County Bridge Commission and Robinson and Steinman, Engineers, November 1938, (unsigned copy)

30 Letter from Eric N. DeLony, Chief and Principal Architect, Historic American Engineering Record, to John P. Isenbarger, Director, Indiana Department of Highways, May 10, 1988.

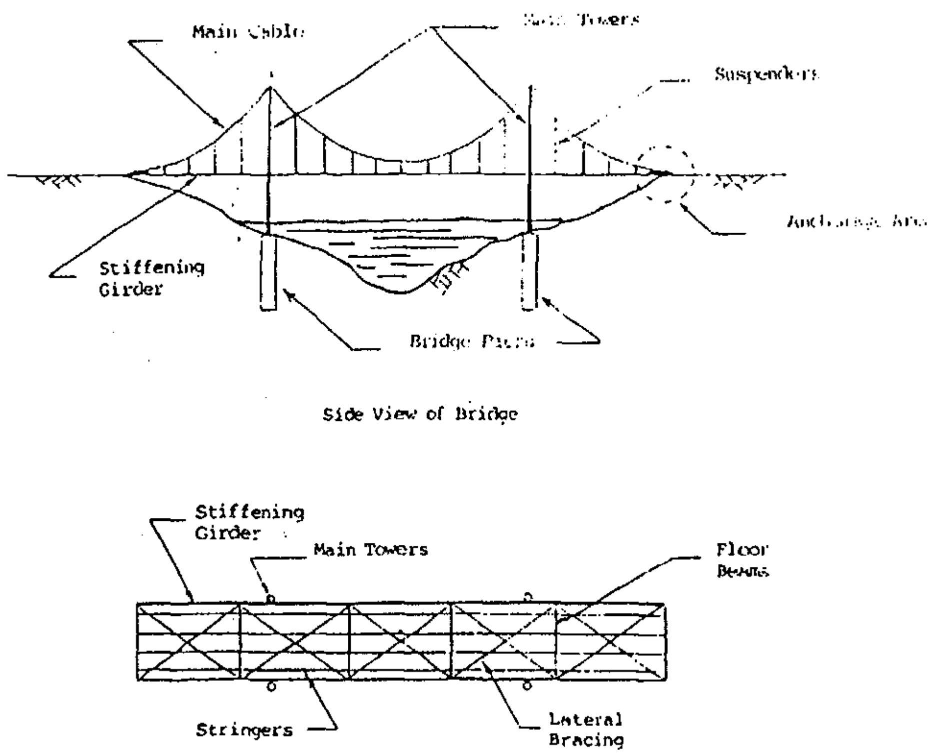
31 Sullivan-Hutsonville Bridge," Souvenir, Sullivan Union Print (circa November 1939), 16 pages

32 Letter from S. R. Osborn and R. M. Kottlowski, American Society of Civil Engineers, to D. L. Klima, Chief, Eastern Division of Project Review, Advisory Council on Historic Preservation, July 11, 1988.

NOTE: Copies of endnotes 1-6 and 29-32 are located in the project files of the Indiana Department of Highways, Division of Location and Environment. Endnotes 7-21 and 28 are located in the project files of the Contracts and Legal Division, Indiana Department of Highways. Endnotes 22-27 are on microfilm in the Newspaper Section of the Indiana State Library.

Self-anchorage vs. External Anchorage
Systems of Suspension Bridges

The basic mechanics of a suspension bridge consist of the transmission of weight from the deck structure (dead load) and from cars, trucks, etc. (live loads) through the suspenders to the main cables. The main cables are supported by twin towers and are anchored at the ends. The towers which support the cables transmit the load from the cables and from their own weight down to the bridge piers. See Figure 1, below.



Side View of Bridge

Top view of floor system

Figure 1 - Suspension Bridge

The two types of anchorage systems are the external and self-anchored system. An external anchorage system uses a large masonry anchor, or other suitable anchor, at the ends of the main suspension cables to counteract the entire tensile force present in the cable. In comparison, a self-anchored suspension bridge uses the stiffening girders in the bridge to counteract most of the tensile force in the cable, the remainder of which can normally be carried by a simple bridge support.

Newton's Law states that for every action there is an equal and opposite reaction. For our purpose, the actions we will consider are tension and compression forces. Tension is a pulling force, whereas compression is a pushing force. The cables in a suspension bridge are comparable to a string and can only transmit tensile forces and not compression forces. The steel girders in the bridge are able to transmit either compression or tensile forces. Both types of forces can be separated into equivalent horizontal and vertical forces, as shown in Figure 2. Force "A" is separated into an equivalent horizontal force "B" and a vertical force "C".

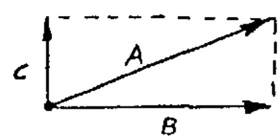


Figure 2 - Force Diagram

An external anchoring system uses a large anchor to counteract the tensile force in the main cable (T_c). See Figure 3, below.

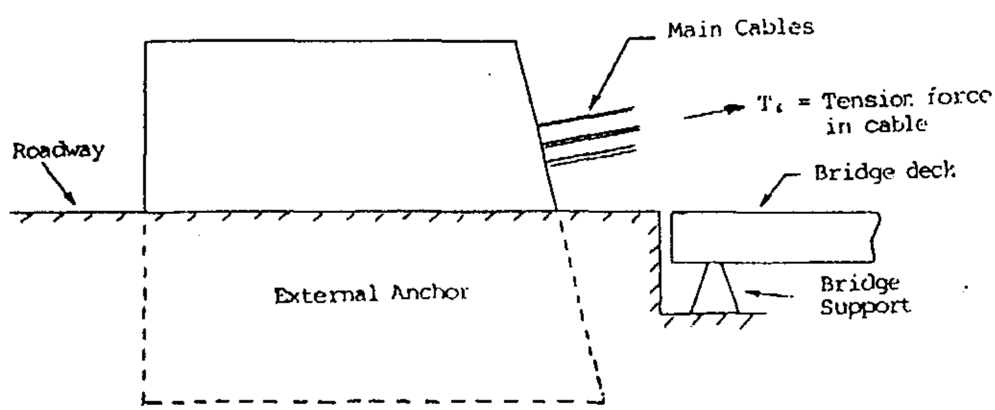
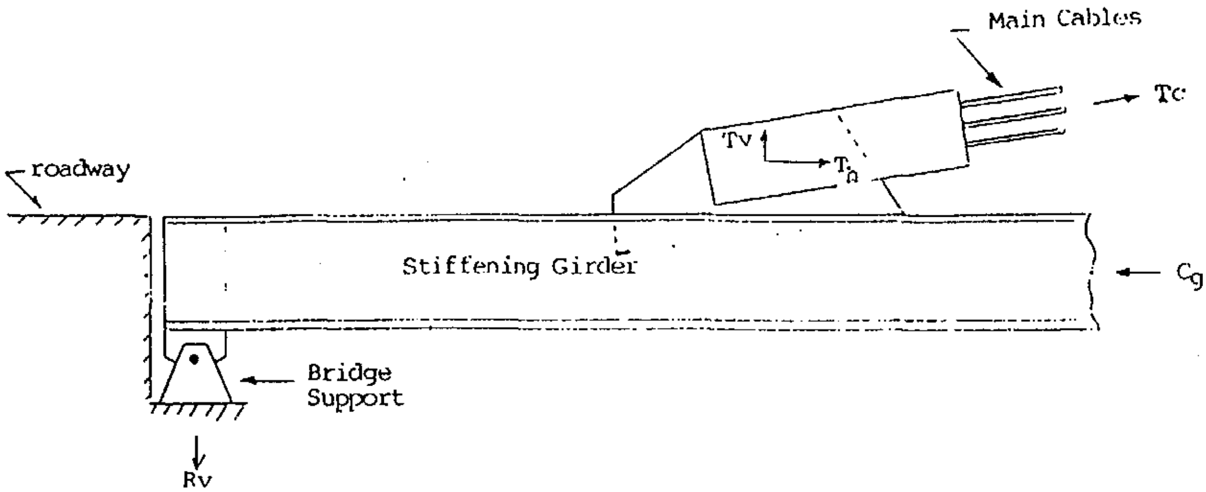


Figure 3 - External Anchoring System

A self-anchoring suspension bridge counteracts the horizontal component (T_h) of the tension in the main cable (T_c), by an equivalent compressive force in the stiffening girder (C_g) [see Figure 4]. Due to the orientation of the cable, the horizontal component (T_h) of the tensile stress is much greater than the vertical component (T_v). The vertical component of the tensile stress (T_v) is counteracted by a reaction at the support (R_v). The reaction needed at the support is small enough that a simple bridge support can be used instead of a large external anchor.



T_c = Tensile force in cable.

T_v = Vertical component of tensile force in cable.

T_h = Horizontal component of tensile force in cable.

C_g = Compressive force in girder.

R_v - Vertical reaction at the support

Figure 4 - Self-Anchoring System

NOTE: The following item is a retyping of an article that first appeared in the June 20, 1940, issue of the Engineering News-Record. It has been reproduced in this manner due to the poor quality copies allowed by the original periodical that is located in the Mathematics Library of the University of Illinois in Champagne, Illinois. A reproduction of the original is also attached.

June 20, 1940 Engineering News-Record (Vol. p. 849)

SELF-ANCHORED I-GIRDER SUSPENSION BRIDGE

C. H. Gronquist
Robinson and Steinman, Consulting Engineers, New York, NY

Contents in Brief - Rolled Beam stiffening girders and multiple twisted wire strands are combined to form a simple self-anchored suspension bridge. Its 20-ft. concrete-filled steel grid deck will carry traffic across the Wabash River south of Terre Haute, Indiana.

Featured by a 350-foot main span, open strand cable construction and continuous girders, the two-lane bridge completed recently over the Wabash River at Hutsonville, Illinois, is the fifth suspension bridge of the self-anchored type in the United States and the longest span in which single rolled beams have been employed as stiffening girders. Throughout their entire length the girders were erected 1 foot above dead load position to facilitate connection of the suspenders, and were then lowered uniformly to introduce the dead load stress into the cables and girders. The bridge, 30 miles below Terre Haute, Ind., and the only crossing in the 60 miles between Terre Haute and Vincennes, will carry interstate traffic across the Wabash River.

A varying depth of sand which amounted to 75 ft. on the Indiana end of the suspended spans, dictated the use of a self-anchored in preference to an externally-anchored suspension structure. The 350-ft. main span is flanked by two 150-ft. suspended side spans and a 352-ft. continuous girder viaduct on the Indiana side to give an overall length of bridge of 1,002 ft. The design of the bridge does not permit provision for movement in the suspended span so all expansion in the 1,002-ft. length is taken in a single joint at the junction between the suspended spans and the viaduct.

The nearness of one end of the bridge to a town necessitated the use of a 5 per cent grade in the side spans and a corresponding parabolic camber in the main span to obtain an underclearance of 17 ft. above the 1913 high water and 42 ft. above mean low water at the center of the main span. The Indiana viaduct, the abutment for which is set in the river levee, is level except for the outer end where a vertical curve merges with the 5 per cent grade on the side span.

The bridge deck consists of a 20-ft. lightweight 4 1/2-in. concrete filled steel grid roadway and two 2-ft. concrete walkways. Concrete for the floor was placed on plywood forms without the use of the customary galvanized steel pans. The steel floor is of typical framed-in stringer and floor beam construction in 25-ft. panels and with the cables and stiffening girders placed 25 ft. apart. All steel in the bridge is structural carbon steel except the stiffening girders and floor beams, where silicon steel is ueed.

The dead load on the suspended spans ie 2,660 lb. per linear ft. of bridge in the main span and a 2,860 lb. per linear ft. in the side spans. Floor members were designed for H-15 live loading while the stiffening girders and cables were designed for a total. live load of 50 lb. per aq. ft. of roadway. The cables, which are streesed to about 67,000 lb. per sq. in. for dead plus live load, are made up of nine 1 1/2-in. twisted wire strands, each of 270,000 lb. ultimate strength, arranged in open, equare grouping. A single 1 5/8-in. wire rope forms the euspender which occurs at every floor beam. Connection of suapender to girder was obtained by elotting the top flange of the girder and inserting vertical platee which were riveted to the girder web and to which the suspender socket was pinned. These plates were welded all around to both sides of the girder flange.

At the anchorage, that ie the connection between the ends of the cable and the stiffening girders, the cable etrand sockets bear through splay collars against the short leg of 8 x 2 x 1-in. angles riveted to the outer end of four 24-in. strand plates which are tied together by plates welded in the field after erection, and at the other end are pin-connected to a bracket in the bridge's stiffening girder.

This bracket occurs at the first panel point from the abutment to avoid complications in the detailing of the end of the etiffening girder other than that produced by the insertion of a deep web plate to form a part of the brAcket and the building up of the first panel length of the stiffening girder. Anchorage of the cable to the girder at the firet panel point subjects the end panel length to additional moment and shear produced by the upward component of the cable reaction. The girder was cambered downward 5/8 in. in accordance with the dead load value of that moment.

In the side spans the silicon steel girders consist of 36-in. wide-flange 300-lb. and 280-lb. beame; in the main span 260-lb., 240-lb., and 230-lb. beame were used. The girder sections were in general 50 ft. long between milled splices designed to develop 100 per cent of the section of the smaller of the girdere at the splice. Splices occurred five feet beyond panel points.

The 6 x 3 1/2 x 3/8-in. angle laterals were designed to function ae a tension system to take 2 per cent of the total dead plua live load direct thrust on the two girdere. The capacity of the laterals so designed is more than sufficient to take the wind shear at the end of the main span, which, however, will be largely carried by the steel grid floor.

Rocker construction was employed for the main towers as well as the Indiana end bent. A 67-ft., 36-in. 182-lb. wide-flange beam forms the leg of the main tower, while two 14-ft., 18-in. channels are used in the end bent. Below the roadway all towers and bents are K-braced; above the roadway portal strut in the main tower the bracing is arranged in the form of the cross of St. George.

Fabrication and erection

The stiffening girder sections were fabricated in straight segments to the main span parabolic camber for dead load and with their lengths corrected for dead load shortening and difference between measuring temperature and the assumed normal temperature of 55 degrees F. Check of girder layout was obtained by measuring assemblies in the shop of half main span length.

The girders were erected on timber falsework bents supported on timber pile clusters set at an averAge of 50 ft. apart. Erection started at both ends of the suspended spans, and closure was made in the main span near the Indiana tower. All steel except the handrail was erected in one pass. The girders were set one foot above dead load position throughout their length, and their riveting was completed before they were lowered. Erection of the cable strands, on which tower positions had been marked at the time of measurement under full dead load stress, was accomplished with the use of timber gallows frames set on top of the towers. Following the erection and slight adjustment of the strands by placing steel shims between the sockets and the splay collars the cable bands and suspenders were placed.

Jacks for lowering the girders were placed at the towers, girder ends, and on intermediate bents in each span. Keeping the lashing which held the girders longitudinally at the Illinois or west abutment taut at all times, the south girder over its entire length was first lowered 1/2 in. at a time, a total of 2 in. The north girder was then similarly lowered a total of 4 in., and progressive lowering of both girders was carried out in the same way for the total 12 in. at the towers and girder ends. Pulling by use of turnbuckles was required to lower the girders at the towers and girder ends, though the pulling at the towers was slight.

After the rocker pins had been driven and the spans swung, the 2-in. downward deflection of the side spans and the 18-in. upward deflection of the main span with respect to full dead load position were in good agreement with the computed values of these deflections. The placing and welding of the steel grid floor, followed by concreting, were done first in the side spans and last in the main span. Supports were used under the girders in the side spans, though that precaution was probably not necessary.

Main Piers

The main piers each consist of two cylindrical shafts connected at the top by an arched concrete strut and set on square bases which were placed as seals within steel sheetpile cofferdams. The Illinois main pier bases were carried to shale; those of the Indiana main pier are supported on steel H-piles driven to shale. The Indiana end pier, which is similar in construction to the main pier, is also supported on steel H-piles driven to shale, while the Illinois abutment, well back from the river, is founded at a depth of 12 ft. in sand and fine gravel. This abutment, of unusually heavy construction, holds the entire suspended structure against longitudinal motion.

The Indiana viaduct consists of seven 50-ft. continuous girder spans in which the bottom flange of the 33-in., 125-lb. wide-flange beam girder is aligned with the bottom flange of the suspended span girder to preserve that line unbroken throughout the length of the bridge. The cross-section of the viaduct is identical with that of the suspended spans except for the difference in the depth of girders. Twelve inch steel H-piles driven to shall and K-braced above ground constitute the viaduct bents, which will bend with temperature expansion or contraction of the viaduct. At two of the bents near the outer end of the viaduct, where the temperature motion is considerable, slotted bronze plates on the girders permit a limited motion by sliding before bending the bents. The outer end of the viaduct girder slides on a bracket attached to the suspended span and bent.

Construction of the Sullivan-Hutsonville Bridge was commenced in January and was completed in December, 1939, at a cost of $200,000 as a toll project of Sullivan County, Ind. The general contractor was the Wisconsin Bridge and Iron Co. of Milwaukee, Wis., for whom Charles J. Glasgow of Detroit, Mich., was subcontractor on the construction of the floor and miscellaneous work. All steelwork was fabricated by the Vincennes Steel Corp. of Vincennes, Ind. The bridge was designed and its construction supervised by Robinson and Steinman, consulting engineers of New York, N.Y., for whom R. V. Milbank was resident engineer. The writer in general charge for the consulting engineers.

ENGINEERING NEWS-RECORD

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F. E. SCHMITT AND WALDO G. BOWMAN, *Editors* • ALBERT E. PAXTON, *Manager*

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THIS WEEK

● Stability at high levels characterized the construction cost picture during 1939. This week, *News-Record* presents its annual resume, 75 pages packed with informative work data for engineering and contracting forces concerned with planning, bidding, appraisal and valuations, in addition to its regular editorial content. This section covers wide variety of cost data—railroads, public utilities, roadbuilding; housing costs—municipal, USHA, federal; material prices, bond interest and money rates, labor equipment rentals, unit bid prices, etc.

● From laminated timber arches for buildings to a self-anchored I-girder suspension bridge, this issue carries a wealth of additional material of vital interest.

● Taking the laboratory to the samples—a new idea in conducting field tests inaugurated by the U. S. Public Health Service on the Ohio River pollution survey—is the subject of an instructive article.

● Historic Austin Dam Rebuilt—a brief pictorial and descriptive record vividly portraying the five stages in its development that have followed recurring failures since the original was built in 1893. The new Austin Dam is integrated in a system of four dams designed for flood-control and development on the Colorado River of Texas.

THINGS TO COME

LIFTING a River over a Levee—an old problem solved in a new way. Departing from conventional practice U. S. Army Engineers installed a siphon to replace a sluiceway on the St. Francis River project. By extensive research on siphon hydraulics a structure was designed with a discharge coefficient of 0.97 as contrasted to the previously established value of 0.75. Hydraulic, structural design and performance data will appear in the forthcoming July 4 *News-Record.*

THE GOOD OLD DAYS of contracting were before the World War, according to E. L. Copeland, of Bates and Rogers Construction Co. His reminiscences of 35 years in the business will be published in an early issue.

tendency to push outward into the street and disturb utility lines. In the pile clusters under footings, the very close spacing made it desirable to first drive the piles around the perimeter of the cluster to prevent heaving. Although many of the piles were as much as 22 in. in diameter, and the 3-ft. spacing left only 14 in. between outer edges of the larger butts, yet no heaving occurred and excellent alignment was attained.

Specifications required the piles to have a minimum diameter of 7 in. at the lower tip. For piles whose lower tips were to go into splices, the minimum diameter was 10 in.

Splices were made by dressing down adjoining ends so that a steel pipe casing of 10-in. inside diameter would fit readily over both ends. A 1x18-in. steel dowel was driven half its length into the lower pile, using a drilled hole $\frac{11}{16}$ in. in diameter to give a hard-driving fit. Around the dowel was placed a 4-in. grid connector which would embed itself to equal depths in the two abutting ends when the squared faces came together. All pile tops were cut off square.

The job was done under the control of the Federal Works Agency, Public Buildings Administration, Geo. P. Hales, construction engineer. The special drivers were designed and built by the firm that held the piledriving contract, Ben C. Gerwick, Inc.

Self-Anchored I-Girder Suspension Bridge

C. H. Gronquist
Robinson and Steinman, Consulting Engineers, New York, N. Y.

***Contents in Brief**—Rolled beam stiffening girders and multiple twisted wire strands are combined to form a simple self-anchored suspension bridge. Its 20-ft. concrete-filled steel grid deck will carry traffic across the Wabash River south of Terre Haute, Ind.*

Featured by a 350-ft. main span, open strand cable construction and continuous girders, the two-lane bridge completed recently over the Wabash River at Hutsonville, Ill., is the fifth suspension bridge of the self-anchored type in the United States and the longest span in which single rolled beams have been employed as stiffening girders. Throughout their entire length the girders were erected 1 ft. above dead load position to facilitate connection of the suspenders, and were then lowered uniformly to introduce the dead load stress into the cables and girders. The bridge, 30 miles below Terre Haute, Ind., and the only crossing in the 60 miles between Terre Haute and Vincennes, will carry interstate traffic across the Wabash River.

A varying depth of sand which amounted to 75 ft. on the Indiana end of the suspended spans, dictated the use of a self-anchored in preference to an externally-anchored suspension structure. The 350-ft. main span is flanked by two 150-ft. suspended side spans and a 352-ft. continuous girder viaduct on the Indiana side to give an overall length of bridge of 1,002 ft. The design of the bridge does not permit provision for movement in the suspended span so all expansion in the 1,002-ft. length is taken in a single joint at the junction between the suspended spans and the viaduct.

The nearness of one end of the bridge to a town necessitated the use of a 5 per cent grade in the side spans and a corresponding parabolic camber in the main span to obtain an underclearance of 17 ft. above the 1913 high water and 42 ft. above mean low water at the center of the main span. The Indiana viaduct, the abutment for which is set in the river levee, is level except for the outer end where a vertical curve merges with the 5 per cent grade on the side span.

The bridge deck consists of a 20-ft. lightweight $4\frac{1}{2}$-in. concrete filled steel grid roadway and two 2-ft. concrete walkways. Concrete for the floor was placed on plywood forms without the use of the customary galvanized steel pans. The steel floor is of typical framed-in stringer and floorbeam construction in 25-ft. panels, and with the cables and stiffening girders placed 25 ft. apart. All steel in the bridge is structural carbon steel except the stiffening girders and floorbeams, where silicon steel is used.

The dead load on the suspended spans is 2,660 lb. per linear ft. of bridge in the main span and 2,860 lb. per linear ft. in the side spans. Floor members were designed for H-15 live loading while the stiffening girders and cables were designed for a total live load of 50 lb. per sq.ft. of roadway. The cables, which are stressed to about 67,000 lb. per sq.in. for dead plus live load, are made up of nine $1\frac{1}{4}$-in. twisted wire strands, each of 270,000 lb. ultimate strength arranged in open, square grouping. A single $1\frac{3}{8}$-in. wire rope forms the suspender which occurs at every floorbeam. Connection of suspender to girder was obtained by slotting the top flange of the girder and inserting vertical plates which were riveted to the girder web and to which the suspender socket was pinned. These plates were welded all around to both sides of the girder flange.

At the anchorage, that is the connection between the ends of the cable and the stiffening girders, the cable strand sockets bear through splay collars against the short leg of 8 x 2 x 1-in. angles riveted to the outer end of four 24-in. strand plates which are tied together by plates welded in the field after erection, and at the other end are pin-connected to a bracket in the bridge's stiffening girder.

This bracket occurs at the first panel point from the abutment to avoid complications in the detailing

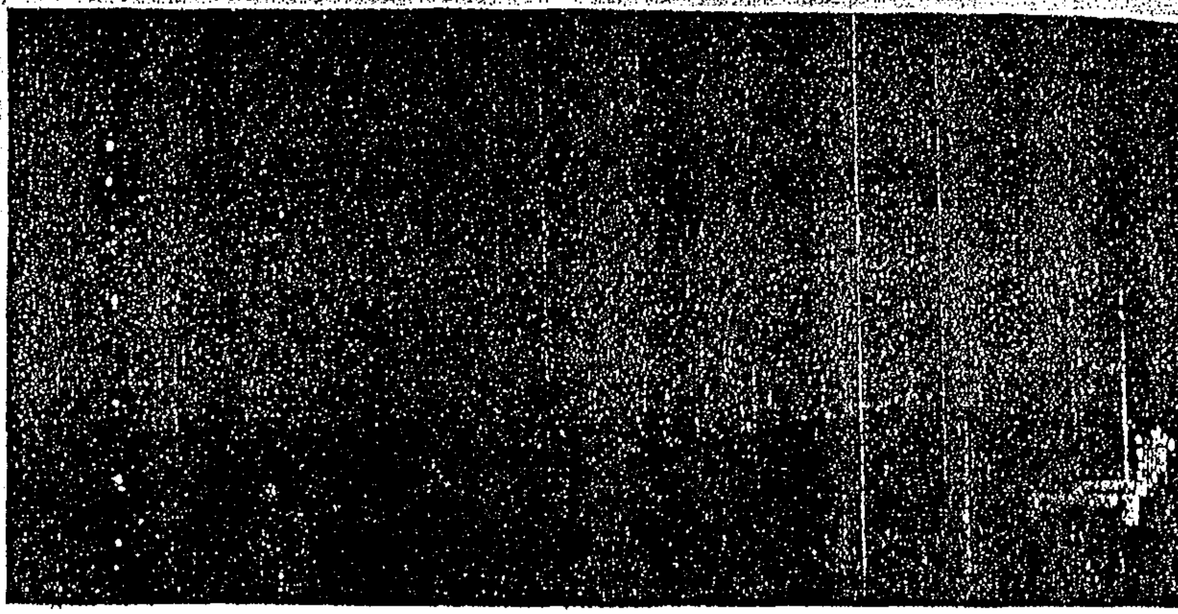

Fig. 1. Rolled beam stiffening girders and cables of twisted strands feature Wabash River self-anchored suspension bridge.

of the end of the stiffening girder other than that produced by the insertion of a deep web plate to form a part of the bracket and the building up of the first panel length of the stiffening girder. Anchorage of the cable to the girder at the first panel point subjects the end panel length to additional moment and shear produced by the upward component of the cable reaction. The girder was cambered downward ⅝ in. in accordance with the dead load value of that moment.

In the side spans the silicon steel girders consist of 36-in. wide-flange 300-lb. and 280-lb. beams; in the main span 260-lb., 240-lb., and 230-lb. beams were used. The girder sections were in general 50 ft. long between milled splices designed to develop 100 per cent of the section of the smaller of the girders at the splice. Splices occurred five feet beyond panel points.

The 6 x 3½ x ⅜-in. angle laterals were designed to function as a tension system to take 2 per cent of the total dead plus live load direct thrust on the two girders. The capacity of the laterals so designed is more than sufficient to take the wind shear at the end of the main span, which, however, will be largely carried by the steel grid floor.

Rocker construction was employed for the main towers as well as the Indiana end bent. A 67-ft., 36-in. 182-lb. wide-flange beam forms the leg of the main tower, while two 14-ft., 18-in. channels are used in the end bent. Below the roadway all towers and bents are K-braced; above the roadway portal strut in the main tower the bracing is arranged in the form of the cross of St. George.

Fabrication and erection

The stiffening girder sections were fabricated in straight segments to the main span parabolic camber for dead load and with their lengths corrected for dead load shortening and difference between measuring temperature and the assumed normal temperature of 55 degrees F. Check of girder layout was obtained by measuring assemblies in the shop of half main span length.

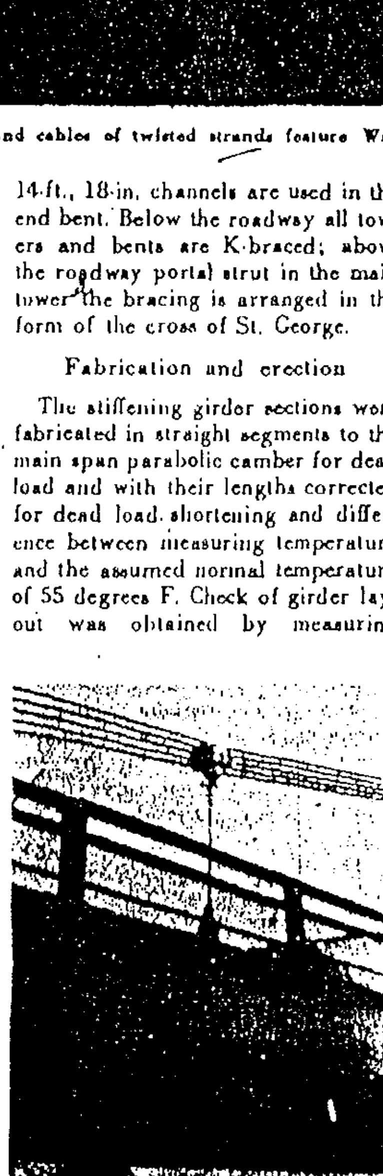

Fig. 2. 1⅝-in. suspender cable and fastenings.

The girders were erected on timber falsework bents supported on timber pile clusters set at an average of 50 ft. apart. Erection started at both ends of the suspended spans, and closure was made in the main span near the Indiana tower. All steel except the handrail was erected in one pass. The girders were set one foot above dead load position throughout their length, and their riveting was completed before they were lowered. Erection of the cable strands, on which tower positions had been marked at the time of measurement under full dead load stress, was accomplished with the use of timber gallows frames set on top of the towers. Following the erection and slight adjustment of the strands by placing steel shims between the sockets and the splay collars the cable bands and suspenders were placed.

Jacks for lowering the girders were placed at the towers, girder ends, and on intermediate bents in each span. Keeping the lashing which held the girders longitudinally at the Illinois or west abutment taut at all times, the south girder over its entire length was first lowered ½ in. at a time, a total of 2 in. The north girder was then similarly lowered a total of 4 in., and progressive lowering of both girders was carried out in the same way for the total 12 in. at the towers and girder ends. Pulling by use of turnbuckles was required to lower the girders at the towers and girder ends,

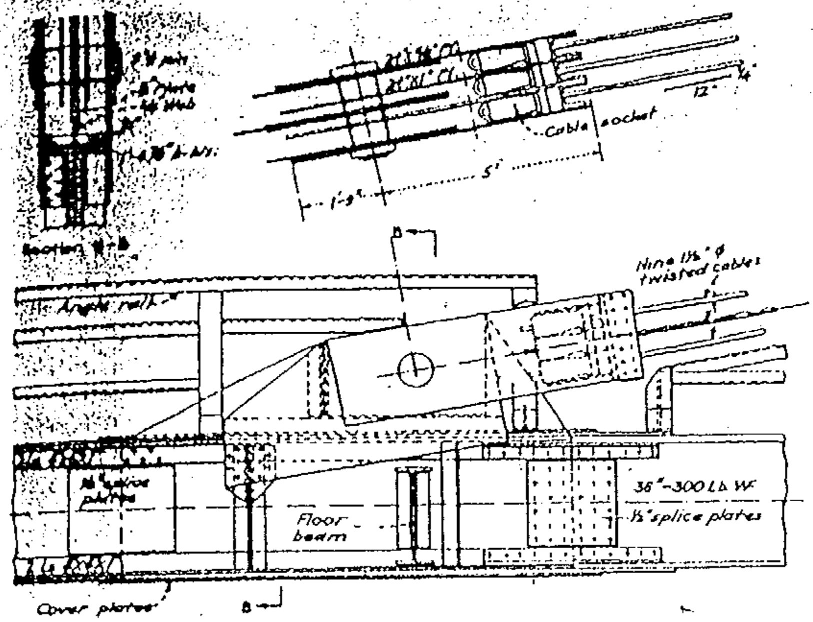

Fig. 3. Anchorage detail of the Wabash River Bridge showing the self-anchoring arrangement.

though the pulling at the towers was slight.

After the rocker pins had been driven and the spans swung, the 2-in. downward deflection of the side spans and the 18-in. upward deflection of the main span with respect to full dead load position were in good agreement with the computed values of these deflections. The placing and welding of the steel grid floor, followed by concreting, were done first in the side spans and last in the main span. Supports were used under the girders in the side spans, though that precaution was probably not necessary.

Main piers

The main piers each consist of two cylindrical shafts connected at the top by an arched concrete strut and set on square bases which were placed as seals within steel sheetpile cofferdams. The Illinois main pier bases were carried to shale; those of the Indiana main pier are supported on steel H-piles driven to shale. The Indiana end pier, which is similar in construction to the main pier, is also supported on steel H-piles driven to shale, while the Illinois abutment, well back from the river, is founded at a depth of 12 ft. in sand and fine gravel. This abutment, of unusually heavy construction, holds the entire suspended structure against longitudinal motion.

The Indiana viaduct consists of seven 50-ft. continuous girder spans in which the bottom flange of the 33-in., 125-lb. wide-flange beam girder is aligned with the bottom flange of the suspended span girder to preserve that line unbroken throughout the length of the bridge. The cross-section of the viaduct is identical with that of the suspended spans except for the difference in the depth of girders. Twelve inch steel H-piles driven to shale and K-braced above ground constitute the viaduct bents, which will bend with temperature expansion or contraction of the viaduct. At two of the bents near the outer end of the viaduct, where the temperature motion is considerable, slotted bronze plates on the girders permit a limited motion by sliding before bending the bents. The outer end of the viaduct girder slides on a bracket attached to the suspended span and bent.

Construction of the Sullivan-Hutsonville Bridge was commenced in January and was completed in December, 1939, at a cost of $200,000 as a toll project of Sullivan County, Ind. The general contractor was the Wisconsin Bridge and Iron Co. of Milwaukee, Wis., for whom Charles J. Glasgow of Detroit, Mich., was subcontractor on the construction of the floor and miscellaneous work. All steelwork was fabricated by the Vincennes Steel Corp. of Vincennes, Ind. The bridge was designed and its construction supervised by Robinson and Steinman, consulting engineers of New York, N. Y., for whom R. V. Milbank was resident engineer. The writer in general charge for the consulting engineers.

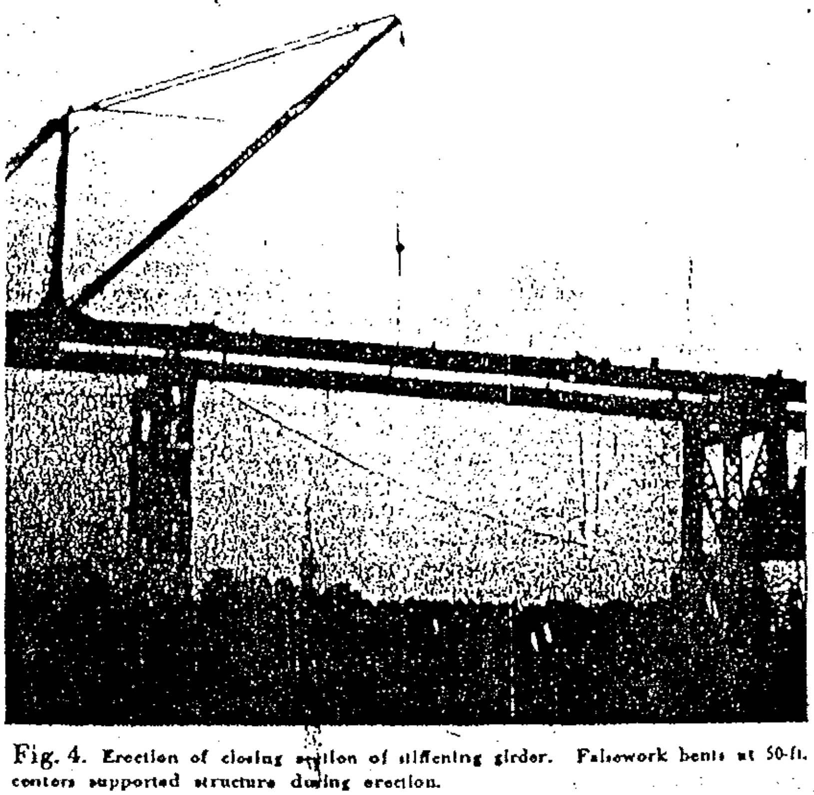

Fig. 4. Erection of closing section of stiffening girder. Falsework bents at 50-ft. centers supported structure during erection.

THE CONSTRUCTION OF A SELF ANCHORING SUSPENSION BRIDGE

A Thesis

Submitted to the Faculty

of

Purdue University

by

Orville W. Eusey

in partial fulfillment of the

requirements for the Degree

of

Professional Engineer

in

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A Thesis

"The Construction of a Self Anchoring Suspension Bridge."

THE CONSTRUCTION OF A SELF ANCHORING SUSPENSION BRIDGE

The completion of a new self anchoring suspension bridge over the Wabash River, connecting Indiana State Road # 154 and Hutsonville, Illinois, fulfills a long felt need for another connecting link between the two states. Picture # 1 is a general view of this bridge. This structure, located approximately half way between Terre Haute and Vincennes, Indiana, where the nearest bridge crossings are located, offers a shorter route between several Southern Indiana cities and St. Louis, Missouri and also provides another outlet for the local coal field.

Authority for the construction of this bridge was created in 1929 by the citizens of Sullivan County, Indiana, with the appointment of a three man Bridge Commission, empowered to engage engineers, sell bonds, and otherwise perform the necessary business transactions.

In 1930, engineers were engaged to design a bridge for a crossing at Merom, Indiana. There was a high bank on the Indiana side of the river at this site, but the approach on the Illinois side was across several miles of very low ground which often was flooded. The Bridge Commission asked the Illinois Highway Commission to co-operate in this undertaking by building an all weather approach road, but because of the excessive cost, they were unable to secure any co-operation from this source, so the proposed crossing at Merrom, Indiana, was abandoned.

After eight years, a more suitable bridge site, with approaches that were above flood water, was found at Hutsonville, Illinois. Here, the west bank was above flood water ~~elevation~~ and the east approach road was protected by a high levee. The plans were revised for this crossing, bonds were sold, and a contract for the bridge construction was awarded in December of 1938.

The Bridge Commission expects to retire the $ 300,000.00 issue of 4½% bonds over a period of 20 years with the toll collected for this purpose. As soon as the bonds are retired, the structure will become a free bridge, and will be turned over to the State Highway Commission of Indiana to become a part of its highway system. Since the State will be responsible for the maintenance of the bridge at this future date, the Sullivan County Bridge Commission ask the State Highway Commission of Indiana to participate in the field engineering, although they had already engaged engineers to design and supervise the construction of this bridge. None of the Bridge Commission members were engineers and they wanted this double assurance that the bridge would be built in accordanee with the best approved engineering principles. This request seemed rather odd at first, but on learning that the Consulting Engineers also invited full co-operation, the State agreed to furnish an engineer at this bridge.

Having had nine years experience as project engineer on various types of steel and concrete bridges, I was chosen as the State's representative for this job, and armed with a roll of bridge plans, I reported at the bridge site on a very cold winter day in January 1939.

It was easy to locate the bridge site as shown on the plans, but it required an elastic imagination to visualize the lines of the graceful bridge which was to rise, where then ~~was~~ were only weeds and underbrush.

I made a general inspection of the site and found the main channel of the river was approximately 400 ft. wide at this point, with the main current and deepest part of the channel near the west bank which was higher than normal flood water elevation. East of the channel was a low bank rising about 10 ft. above low water elevation and extending back about 500 ft. to the toe of a high levee which had been built to keep flood water from covering about 25 square miles of low lying farm land.

Looking over the plans, I learned the bridge designed for this crossing was a new type of structure. It was a self anchoring suspension bridge, consisting of a 350 ft. main span and two 150 ft. anchor spans. The East approach to the suspension spans consisted of seven, continuous, steel girder, viaduct spans of 50 ft. each.

The viaduct spans were to be built on a level grade from the east abutment which was at the levee, to the east anchor span pier which was on the low, east bank about 120 ft. from the low water channel. From this point, the east anchor span was to rise on a plus 6% grade to the main span, which was to be built to a parabolic curve between this plus 6% grade and a minus 6% grade of the west anchor span.

The bridge floor was 20 ft. wide between curbs with a narrow sidewalk on each side. This width of roadway is approved by the A.A.S.H.O. for class "A" Bridges.

The loads used in the design were an H-15, live load for the viaduct spans and 1000 pounds per linear foot, live load for the suspension spans. A wind load of 200 lbs. per linear foot was also considered in the design, and special wind connections were provided at the towers and also at the ends of the suspension span.

The suspension span of this bridge had no concrete anchorages, but was designed so the horizontal component of the cable tension was taken by axial compression in the stiffening girders. Besides having this unique characteristic of being self-anchoring, this structure had several other features which made it of unusual interest to an engineer. A few of these features are: the use of 94 ft. steel H-beam piles; the use of thin, concrete filled, I-Beam floor; and the use of a single expansion joint for the 1000 ft. of steel superstructure. But these features will be described in their proper order, as it is better that a description of this bridge begin at the bottom, which would be the test holes 63 ft. below the water level.

Before going into a description of the construction of this suspension bridge, I will outline the part I played in its construction.

A project engineer's principal duty, besides actual field surveying in laying out the work, is to inspect all parts of the construction as it progresses.

I assisted in all the surveying at this bridge; and in checking the elevations which the designing engineers had used, I found their field party had made an error of about 3 ft. in using a government bench mark. Consequently, elevations at the bridge were revised; and it was found that the main span could be lowered and still have the required clearance above low water elevation, and the grade of the anchor spans could be reduced from a 6% grade to a 5% grade, thus improving the appearance of the bridge, and making it safer to the driving public by increasing the sight distance.

The surveying at this job involved precise measurment of distances. All measurments were corrected for temperature, sag, and tape tension. The main span across the channel was triangulated and then checked by a piano wire measurment.

In general, my duty as inspector was to see that all parts of the structure were built in accordance with the best approved engineering principles. I investigated subsoil conditions and suggested changes necessary for securing suitable foundations, passed judgment on means and methods by which the contractor proposed to carry on the work, and suggested changes which I thought would result in a better structure. I was to accomplish all this through diplomatic means in so far as possible; but failing, I was to report to my superiors, who would then take the matter to the Bridge Commission. I received full co-operation in all my work and no such action was necessary.

My first job on this structure was to inspect the material taken from the test holes and make a log classifying the different materials encountered in drilling these holes.

Satisfactory test holes were drilled by a small portable rig equipped with a light drop hammer and rotary drilling machinery. A casing was driven from 3 ft. to 20 ft. and then washed out by a pressure pump mounted on a rig. Very few dry samples were taken because much of the material was a coarse sand or fine gravel and would not hold in the sampler. All washings were examined closely and the log was made from these observations. On reaching rock, the rotary drilling machinery was fitted with a diamond pointed, core drill, and from 3 ft. to 17 ft. cores were taken. A firm shale was found under the west abutment and west pier, at 5 ft. and 25 ft. below low water elevation. Under the other foundations the shale was 59 ft. to 63 ft. below low water elevation.

After a careful study of the boring data, it was evident to me that ten of the foundations were unsatisfactory as designed, because they were founded in very sandy material and the proposed elevation for the bottom of these foundations was as much as 15 ft. above the lowest elevation of the main channel, where any shifting of the channel would undermine them and cause complete failure.

The west abutment foundation was satisfactory as designed since it was founded on a strata of sandy gravel with sufficient bearing capacity for carring the load, and was located back from the river where there was no danger of it being undermined.

I suggested changes for the other ten foundations and the Bridge Commissioner's engineer readly agreed since the original design had been made before the boring data was available.

The following is a brief description of the foundations as first designed and the changes necessary to secure satisfactory foundations.

The main pier footings were concrete blocks, 14 ft. square and twelve ft. deep, under each leg of the pier. The west main pier footing was lowered 2 ft. and decreased to a block, 12 ft. square and 14 ft. deep. This keyed the footing about 2 ft. into a firm shale, and made a very satisfactory foundation. The east main pier footing was lowered 5 ft. as a further protection from undermining, and sufficient steel H-Beam piles (nine under each footing) were added to carry this pier should it be fully undermined. Steel H-beam piles also were added under the anchor pier and the east abutment.

All the steel, viaduct pier bents, originally designed to set on small concrete pedestals, were revised. Since the footings for these concrete pedestals were very small, it was impractical to place them on piles; and because it would have been very costly to increase their size and extend them below danger of undermining, these footings and pedestals were eliminated and the steel pier bents were redesigned as a pile bent consisting of two, 70 ft., steel, H-beam piles framed together above the ground. A small concrete jacket was placed around each pile near the ground line as a protection against rust, because it is in this area that deterioration usually begins. (Picture # 1 shows the steel pile bents and concrete jackets very distinctly.)

From my observations of the driving of the steel casings used in the test boring, I suggested that all steel piling would have to extend to rock to secure the necessary bearing capacity. This suggestion was carried out in the two heavier foundations, but the Bridge Commissioner's Engineer thought we might be able to secure the necessary bearing by using 70 ft. piles for the viaduct bents and 50 ft. piles for the east abutment foundation. Later, it was necessary to add 24 ft. to each pile and drive them to rock as I had suggested, because only one third the necessary bearing capacity was developed with this shorter length of pile.

These changes in design were worked out while the contractor was unloading and rigging his equipment, and he was able to begin actual construction late in January 1939.

The original plan of operation was to open up the main pier foundations first, using stiff leg derricks on each bank. The derrick on the west bank was to complete the west main pier and abutment, and handle structual steel on the west bank. The derrick on the east was to complete the east main pier, the anchor pier, drive the viaduct bent piles, and then complete the east abutment. This derrick then was to be placed on top and used to erect the struotural steel. This original plan, as on most construction jobs, was changed for two reasons. First, the river began to rise; and second, the delivery point for the structural steel and the order of erection was changed.

Excavation for the west main pier was started January 25,1939. The river bank was leveled off just above water elevation so a cofferdam frame could be set up as a guide for the steel sheeting. Before the frame could be set,the river began to rise;and in order not to delay the work, timber piles were driven outside each corner of the foundation, the cofferdam frame was supported on these piles above the rising water,and the steel sheet piles were set up around the frame and driven a short distance. (Picture # 2 shows the frame in place ready for the steel sheeting in one leg of the west main pier foundation and the sheeting already in place in the other leg.) Additional framing was added and driven down as the excavation progressed. The steel sheeting was driven ahead of the excavation,holding a minimum toe-in of about 5 feet. When firm shale was encountered at an elevation 2 ft. above the proposed bottom of the foundation, it was broken up by dropping a pointed, 60 ft. section of railroad rail into it, and was removed by a clam bucket on the derrick. After most of the material had been removed by this method , a diver was employed to complete the excavation. (Picture # 3 shows the diver going to work.) Working in 30 ft. of water, this diver used a jackhammer to loosen and level the material in the bottom of the foundation. Most of his work was around the edge of the cofferdam and under the framing where the pointed rail could not be used. After loosening the material, he pulled it to the center of the hole where it could be removed by the clam bucket. A very satisfactory foundation was secured in this manner.

The cofferdam was not unwatered until after the footing was poured so I inspected this foundation by sounding over the area with a steel rod and making visual inspection of the last material removed from the hole. No test boring was required as one of the original test holes was drilled in the area of this small foundation.

Excavation for the east main pier foundation was made in the same manner as outlined above, except it was not necessary to use a diver as there was no shale in this foundation. Instead, 49ft., steel, H-beam piles were driven to firm shale under this footing. The bearing capacity of the sandy material on which this footing rested was sufficient to carry the load safely, but H-beam piles were added to carry the entire load should the foundation be fully undermined. (Picture #4 shows a few of these piles partially driven.)

Excavation for the abutments was relatively shallow and offered no particular difficulties. Both abutments were well back from the channel; but as the excavation for the east abutment foundation neared completion, heavy snows and rains caused the river to rise, and it was necessary to protect the break in the levee caused by the excavation. Although several methods could have been used, I suggested that complete back filling inside and outside the cofferdam would be the safest way to protect the property behind the levee, because the cofferdam was constructed of light material and was not designed for heavy duty. Also, this method was not costly because the amount of material involved was small, and it was easy to remove the material from inside the cofferdam after the water receded. (Picture # 5 shows the water well up on the levee, and also shows the partially inundated excavating equipment.)

The first steel, H-beam piles were driven in the east abutment foundation. Piles were driven with a number 9-B-2, McKiernan & Terry, steam hammer developing 7000 ft. lbs. of energy per blow. The 50 ft. piles as called for on the revised plans were driven, but only 18 tons per pile bearing capacity was developed. The intent of the design was to develope approximately 50 tons per pile so it was necessary to splice an additional 24 ft. length onto the piles and drive them to hard shale.

It was now evident that the viaduct bent piles would not develop sufficient bearing capacity unless they also were driven to rock as I had suggested when the revision of the foundation plan was made; therefore, an additional 24 ft. length of pile was ordered at this time.

The contractor elected to use a bolted connection, rather than a welded connection, for splicing this additional length of pile. First, the holes in the top section were drilled through a steel templet; the top section of the pile was set in place and spot welded to the section already driven; and then the lower holes were drilled, the splice bolted up, and the nuts spot welded to prevent loosening while driving.

When I inspected the first bolted splice, I found a slight opening between the two sections of the pile where the ends had not been milled square. I had this opening welded full to give perfect bearing between the ends of the piles, and thus reduced the shearing stress on the bolts and prevented any eccentric loading at this point.

No difficulties were encountered in driving the piles. They were plumbed two ways with transits and driven through a timber guide at the bottom while the top was held in position by guy ropes. In the east main pier foundation where the top of the finished pile was below water, angles were welded to the web of the pile near the top to aid in transferring the load from the foundation to the pile. These angles may be seen in Picture # 4. In the east abutment where the top of the piles were above the water, caps were welded on the top of the piles after driving. All foundation piles extended from three to seven feet into the footings, fixing the top in the manner of a fixed column. All the piles, except the abutment piles, were built up on the points to keep the bearing at the points well within the limits recommeded by good practice. (i.e. 3000 to 6000 lbs. per sq. in.) The areas of the bottom ends of the piles were increased from 14.4 sq. inches to a maximum of 47.7 sq. inches, by welding plates and angles on the web and flange of the H-beam section.

An interesting phase of the work was driving the piles before unwatering the cofferdams. After driving all the piles to just above water line, compressed air was blown into the hammer to keep the chamber around the plunger free from water and the piles were then driven to the desired bearing. All piles were driven to practical refusal, thus securing a bearing value of 70 to 85 tons per pile which is the capacity for this size hammer.

All pier footings were poured before unwatering the cofferdams. A 10 inch, watertight tremie pipe, topped with a funnel shaped hopper holding a cubic yard of concrete, was used in pouring this concrete. No foot valve was used on the bottom of the tremie pipe, because after the first charging, the bottom of the tremie pipe was not raised above the freshly placed concrete until the footing was poured. A derrick handled the tremie pipe as well as the bottom dump, concrete bucket. Dowel bars extending into the footings were spot welded into a rigid frame, and lowered between guides to the correct elevation in the fresh concrete as soon as the last concrete was poured. Although these waterseal footings were designed to provide sufficent weight to prevent flotation from 100% hydrostatic pressure with the water level 5 ft. above mean low water, the cofferdams were not unwatered until the concrete had set for seventy-two hours. After unwatering the cofferdams, picks and jackhammers were used to remove the laitance and level the concrete.

The west abutment was a heavy reinforced concrete type with U-type wings. The wings were connected by two tie beams and had a floor across the top at roadway level. The vertical component of the cable pull, in this type suspension bridge, is resisted by the weight of the end supports, in this instance the west abutment; and four, $2\frac{1}{4}$ inch dia. anchor bolts, 12 ft. long, were set in the abutment bridge seat to transfer this uplift from the stiffening girder to the abutment. The uplift due to the dead load amounted to about $22\frac{1}{2}$ tons per girder, while the live load reaction varied from a plus 8.8 tons to a negative or uplift of 30 tons per girder.

The east abutment was a smaller reinforced concrete type, consisting of a heavy cap on two heavy columns with short wings cantilevered back at each end of the cap. The top of the cap was shaped to conform to the roadway and sidewalks. This abutment carried the fixed end of the continuous girder spans.

Each main pier consisted of two, round, reinforced concrete columns connected at the top by a reinforced concrete strut. These columns were 8 ft. in diameter at the footing and 7 ft. in diameter at the top. The lower section of these pier columns was given two coats of Inertol, a water proofing preparation. Picture # 6 is a view of the east main pier.

The anchor pier was similar to the main pier except the columns were square and smaller. These columns were $5\frac{1}{2}$ ft. square at the footing and 4 ft. square at the top. The main function of this pier was to resist the uplift caused by the vertical component of the cable pull.

All forms for concrete were built up in sections at the site and set in place, either by hand or by the derricks.

The forms were made up of $\frac{3}{4}$ inch siding on 2x6 inch studding and were secured in place by 3/8 inch tie rods. Above the ground line, the forms were lined with $\frac{1}{4}$ inch plywood over the $\frac{3}{4}$ inch siding to give a smoother finish to the exposed concrete. Picture #7 shows a section of the pier form ready to be set in place. The plywood lining can be seen near the top. Exposed concrete surfaces were rubbed with a carborundum stone as soon as the forms were removed and were given a final rubbing just before the contract was completed.

The concrete mix was designed by a testing laboratory. Grade "A" concrete was a nominal 1:5 mix containing a minimum of 6.16 bags of cement per cu. yd. and having a compressive strength of 3200 lbs. at 28 days. This class of concrete was used in thin walls and the floor. Grade "B" concrete was a nominal 1:6 mix containing a minimum of 5.64 bags of cement per cu. yd. and having a minimum compressive strength of 2600 lbs. at 28 days. Foundations and heavy walls were poured with this class concrete. Sample cylinders, made for each 250 cu.yds. of concrete as a check on the quality of the concrete, broke as high as 4600 lbs., and all broke well above the required strength. Careful control of the concrete was maintained by weighing all aggregates. The concrete above the water seal was placed through metal tremies and puddled in the forms by mechanical vibrators.

The viaduct bents consisted of two, 94 ft., 12 inch, 49 lb., steel H-beam piles driven about 75 ft. to practical refusal into shale and framed together above the ground by steel struts and braces. The piles were cut off at the correct elevation and a polished bronze bearing plate for supporting the girder was welded on the top of each pile. A vertical pin in this bearing plate fitted into a slotted hole in a similar bearing plate on the bottom of the girder. The hole was slotted so the girder could move longitudinally but could not move tranversely. Any movement of the span due to temperature changes, caused the girder to slide on these bronze bearing plates and thus did not transfer undue bending to the pile bent. (Picture # 8 shows a portion of a bent with the bronze bearing plate welded in place.)

The superstructure of the viaduct spans consisted of seven, 50 ft., continuous girder spans. (Picture # 9 shows a portion of the viaduct during erection.) The main members of each span consisted of two, longitudinal, 33 inch, rolled beam girders, one along each side of the bridge, supported on the pile bents as described above. The transverse floor beams, which were spaced 25 ft. on center, framed into the side of the girders, alternately, over the pile bent and at the center of each span. Four 18 inch stringers between the floor beams carried the floor, which was a 4¼ inch, concrete filled, I-Beam Lok Type.

To an engineer, the main spans of this bridge are more interesting than the substructure or viaduct spans which were a variation of standard type bridge construction.

This bridge had a 350 ft. main span with 150 ft. anchor spans. The stiffening girders were 36 inch, silicon steel beams some of which weighed 300 lbs. per ft., the heaviest rolled section made. The end girders of the anchor spans, to which the cable anchored, were built up of heavy plates and angles because an extra heavy section was required to transfer the vertical component of the cable pull to the abutment through these girders.

The girders were continuous from end to end, and the bottom of the 36 inch girders aligned with the bottom of the 33 inch viaduct girders to present a pleasing, unbroken line from end to end of the structure. (Picture # 1 shows the line of these girders very well. The level section in the foreground is the viaduct spans.) To insure perfect bearing, the ends of all girders were carefully milled and the rivet holes were drilled after the girders had been shop assembled to the correct grade and camber.

The floor beams and stringers were the same as described for the viaduct spans. A suspender from the main cable connected to the girders directly over the floor beams which were spaced 25 ft. on centers. (Pictures # 10 and # 11 are general views of the floor system of the main span before the deck was placed.)

The supension structure was fixed at the west abutment and the viaduct structure was fixed at the east abutment. A sliding joint over the anchor pier joined the spans and all the expansion for the 1000 ft. of steel superstructure was confined to this one point, eliminating troublesome intermediate expansion joints in the roadway. Picture # 12 shows this connection very well. The girder of the suspension span, on the left in the picture and on blocking 1 ft. above its final position, was pin connected to the top of the steel pier bent which rocked on a pin connection at its base thus giving complete freedom for expansion in the suspended span. The sliding connection to the viaduct span is on the right in the picture near the top of the steel bent. A square pin, fixed to the pier bent, slides in a long slotted hole in the plate riveted to the web of the viaduct girder. Polished bronze bushings were used in this connection to insure freedom of movement. A flat plate, fixed down by springs, bridged the gap in the deck at this point.

The towers, shown in picture # 6 and # 10, were 36 inch I-beams, 70 ft. high, framed together above and below the deck. The bottom of the towers rested on rocker plates and the top was fixed to the main cable. Consequently, longitudinal movement of the supension spans merely caused the towers to rock slightly. A special sliding connection was made at the point where the stiffening girders passed through the towers. Picture # 13 shows this connection, which is similar to the expansion connection described above. The connection in the foreground of the picture is known as the tower wind connection and its function is to prevent lateral movement caused by the wind. Similar wind connections were made at the abutment and anchor pier. The sliding connection to the girder, on the right in this picture, is also a part of the wind connection because the bottom flange of the girder is set between plates which prevent lateral movement, and more important the pin connection dampens any vibration in the superstructure caused by wind striking the underside of the deck.

Where the spans are longer this force is more severe and requires special consideration. On a 1200 ft. span recently completed, cables were fastened from the underside of the girders to a lower point on the towers to dampen the vibration caused by the wind striking the underside of the deck. This structure, which is located on the Maine coast, weathered the hurricane of 1939 very satisfactorily.

It is necessary to erect the stiffening girders on falsework in this type suspension bridge, because all the girders must be in place before they can function in axial compression. The falsework required at this structure is shown in pictures # 14 and # 15. The girders were fabricated in 50 ft. lengths and required a bent of falsework every 50 ft. Since the falsework was about 65 ft. high, double deck bents were used. A drop hammer, mounted on a small barge, was used to drive the lower four piles in each leg of the bent. Picture # 17 shows the derrick barge at work. After the lower piles were capped and braced, the upper section was set by the derrick working on top of the steel superstructure. All bents were well X-braced transversely and the top of the bent was secured with cables to prevent longitudinal movement. Picture # 18 shows the derrick setting the first bent. The cables anchoring the top of the bent, and the transverse X-bracing may be seen in picture # 14.

Both anchor spans were erected by derricks working from the ground, and then a small 8 ton derrick, working on top of the steel superstructure, erected the main span. Steel was carried to the derrick on a small barge. Picture # 14 is a view during erection. At this point the derrick was sitting on the portion of the anchor span girder which was cantilevered through the tower and was raising the next 50 ft. section of the girder. A better view of this may be seen in picture # 18. The span at this point was 75 feet. Stresses, caused by the derrick working on top, were investigated for all positions of the derrick. No extra precautions were required as the girder section was sufficient to carry even the above cantilevered set-up with safety.

Stiffening girders were erected one foot above dead load position to facilitate connecting the suspender cables. After all cables were in place, the girders were jacked down, the cables took up the load, and the falsework was removed.

Each main cable at this structure consisted of nine strands of 1½ inch diameter wire rope. This wire rope differed from the usual type cable in that the twist, or lay, of the individual small wire was very long and alternate layers of small wire twisted in opposite directions. Each strand had a gross area of 1.34784 sq. inches and weighed 4.663 lbs. per linear foot. Each strand consisted of 51 wires, varying from 0.100 inch to 0.196 inch in diameter. The specified ultimate strength was 270,000 lbs. per strand.

In order to eliminate some of the stretch from the cable, all strands were prestressed to one half the ultimate strength. Then, in order to have the correct sag in the cable under dead load conditions, the cable was measured and marked while under a stress equal to the dead load. Besides accurately measuring the total length of each strand, the distance to the center of each suspender cable as well as the center of each tower was measured and marked. Also, while under this stress, any tendency for the strand to twist was removed through swivel end connections and a paint stripe was marked on one side of the cable for its entire length so the strand would be erected in this same position and thus would not twist under dead load conditions. (Picture # 19 shows the cable as delivered to the bridge and picture # 22 shows the cable after erection.)

The suspender cable consisted of a single strand of 1 5/8 inch diameter, twisted, wire rope cable, with a gross area of 1.228 inches and weighing 4.363 lbs. per linear foot. Each strand consisted of 133 wires varying from 0.107 inch to 0.128 inch in diameter. The ultimate specified strength was 215,000 lbs. per strand. Each strand was prestressed to 107,500 lbs. and was cut and marked under a stress of 31,700 lbs.

The erection of the cable was relatively simple. A reel was set up on each tower and at the center of the main span, then each strand was pulled across the structure by a small cable attached to a gasoline hoisting engine. Pictures # 11 and # 20 shows the first strand in place over the towers and the second cable being erected. Note the tag line to the lever which the workman is guiding over the reel. This prevented the cable from twisting during erection.

On most wire rope suspension bridges, the strands are layed together and wrapped with small wire after being given a heavy coat of lead paint. But at this bridge the strands were placed in three layers of three strands each with the strands spaced about 4½ inches center to center in both directions. The cable was not wrapped because of the added cost, and also because the maintenance of the open cable would not be difficult on this small bridge. (In picture #22, the open cable shows very distinctly.)

As soon as the three strands of the lower layer were in place, they were adjusted to the correct elevation. This adjustment was accomplished by adding steel shims at the socket at either end of the strand. (The normal 2 inch shim and socket can be seen in picture # 20.) The center of the tower as marked on the cable was first clamped at the center of the tower, and then with the normal shims in place at the end of the strand, four simultaneous readings were taken; the temperature, the sag or elevation of the strand at the center of the span, and a reading at each tower as to the motion of the tower from a plumb position. Then from charts which had been made up previously, the corrections were made by adding shims to vary the length of the strands. The ratio of the change in length to the change in elevation was small in the main span, being approximately 1.82; but was larger in the side spans, being approximately 6.64. Subsequent strands were adjusted by giving them the proper clearance above the lower strands. When all the strands had been placed and adjusted, the suspender cables were clamped to the main cable at the points already marked and connected to the stiffening girders which was on falsework one foot above its final dead load position. Then jacks were placed at all points of support and the stiffening girders were lowered until the cables took up the load. (The completed cable may be seen in picture # 22.)

The floor of this structure was a concrete filled, $4\frac{1}{2}$ inch, I-Beam Lok Type with a 20 ft. clear roadway. Picture # 21 shows a portion of the I-Beam Lok in place before the concrete was poured. Note that the floor does not extend to the side girders, but is carried entirely on the stringers. The concrete was struck off even with the top of the steel I-beams except along each side were an 8 inch curb was raised and a 4 inch reinforced concrete sidewalk was cantilevered over to approximately the center of the girders, but was kept clear of the top of the girders. The curb and sidewalk was poured monolithic. Drains were provided through the floor every 50 ft. This light type floor was adopted to keep the dead load small and therefore the stress in the cables at a low figure.

The handrail consisted of wire mesh on steel angles, supported by 6 inch I-beam posts spaced about 8 ft. on centers. (See picture # 1 and # 22.) The original plans showed the wire mesh continuous over the entire length of the bridge. I suggested that the wire mesh be made up in panels rather than continuous , so any panel which became damaged could be replaced without regards to any other panel. This change was made. The handrail was designed so as not to obstruct the view any more than necessary, and still protect the traffic.

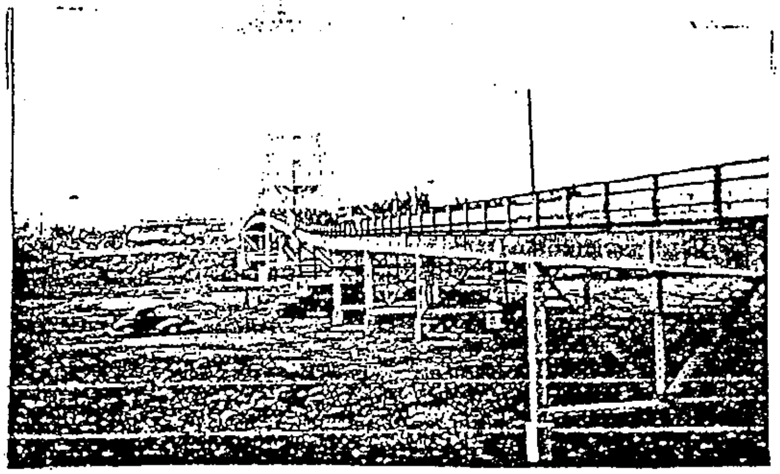
The steel superstructure was given two coats of green paint; primarily, of course, to protect the steel, but the green shade was chosen to better blend the slender towers and the graceful lines of the cables and stiffening girders with the natural beauty of the surrounding countryside. The resulting bridge is not only of definite value to the community, but also a monument to the foresight and enterprise of this community.

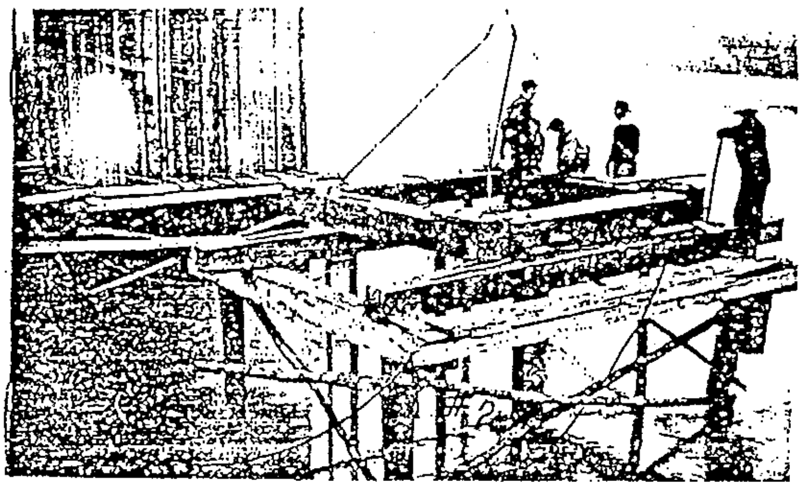
In this connection it might be well to mention that the citizens of this community took a great interest in the construction of this bridge. The so called "Sidewalk Superintendents" were first recognized when observation windows were installed for spectators during the erection of buildings in Rockfeller Center in New York City. While no efforts were made for their comfort at this bridge, visitors were welcome as long as they kept clear of the work. Picture # 16 shows eighteen "Sidewalk Superintendents" on the job.

Construction of this Bridge began in January 1939, and despite much high water during the spring months, it was ready for traffic early in December of that year.

The completion of this self-anchoring span marked the construction of the fifth bridge of this type in the United

States, and the Sixth in the Americas. Three of these bridges are practically identical and are located in Pittsburg, Pennsylvania. The others are small bridges; one located in Missouri over the Little Niangua River, and another on the Pan-American Highway in Guatemala. However, this type bridge has been used extensively in Europe. The Bridge was designed for the Sullivan County Bridge Commission by Robinson & Steinman, Engineers, and constructed by the Wisconsin Bridge & Iron Company. To me, it was a privilege and a pleasure, to have been chosen as project engineer for this job, and to have had a part in the building of a bridge of such an unique design.





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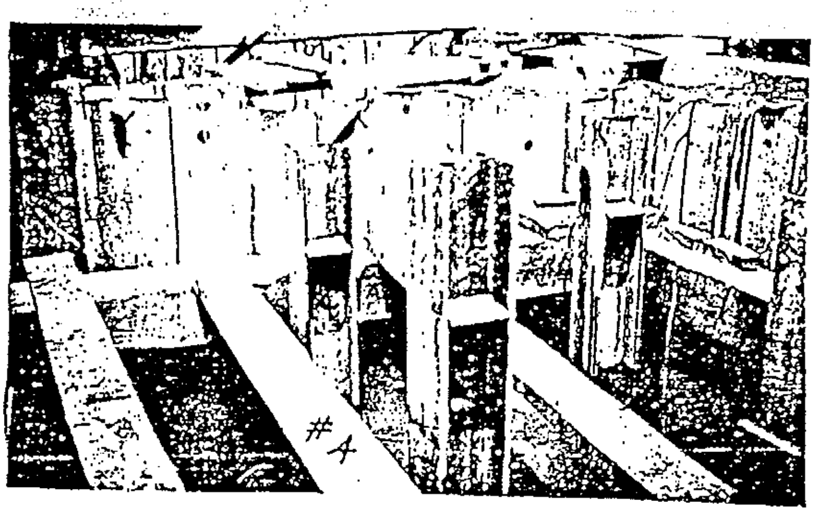
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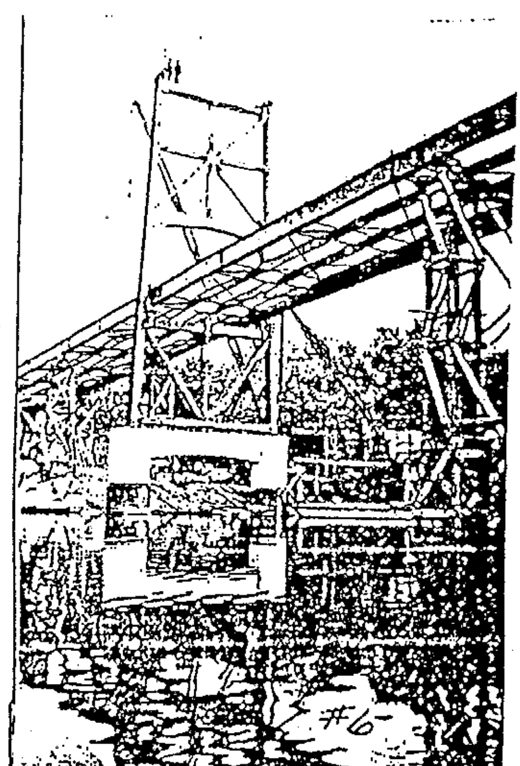
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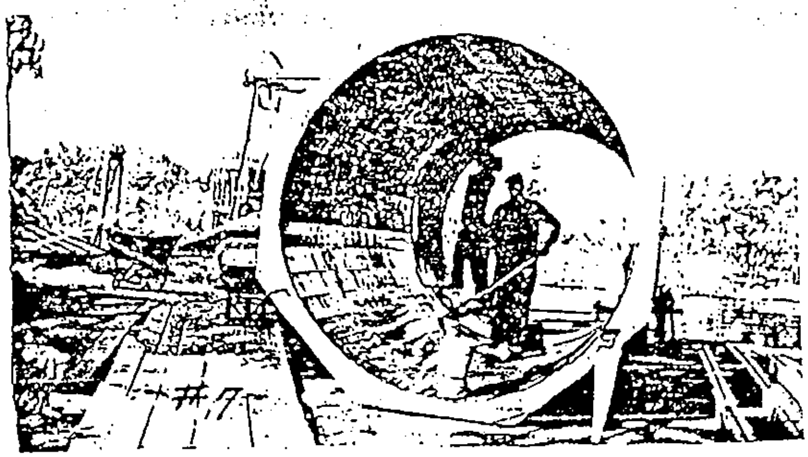

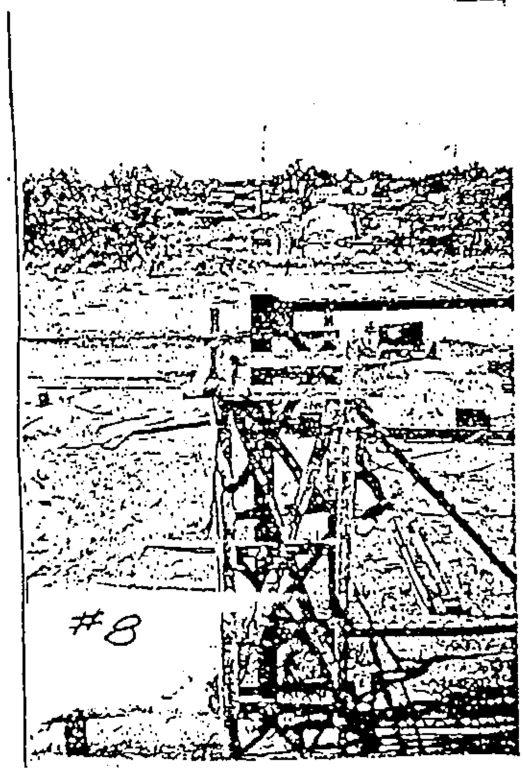

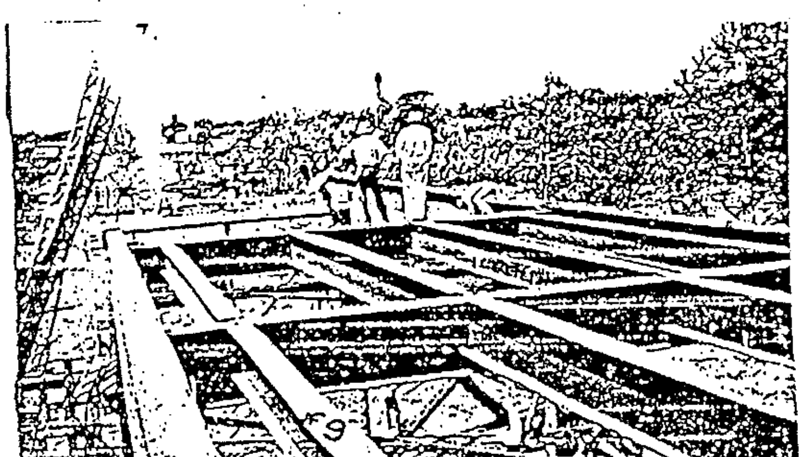

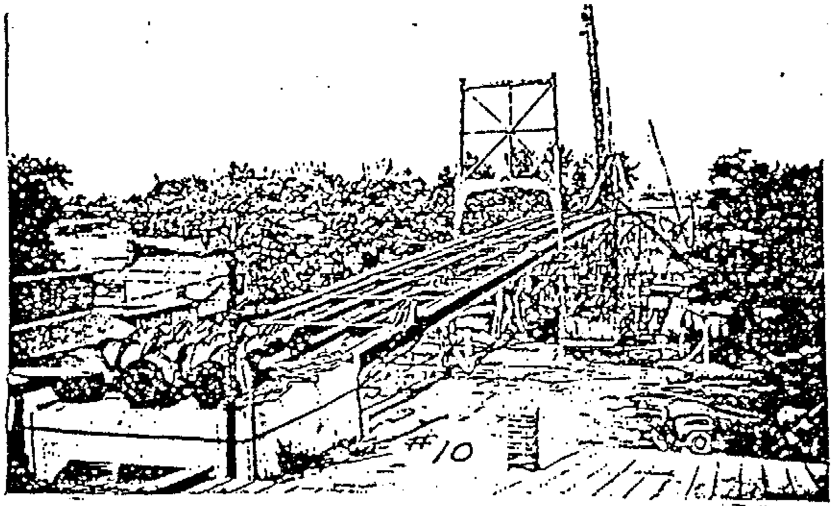
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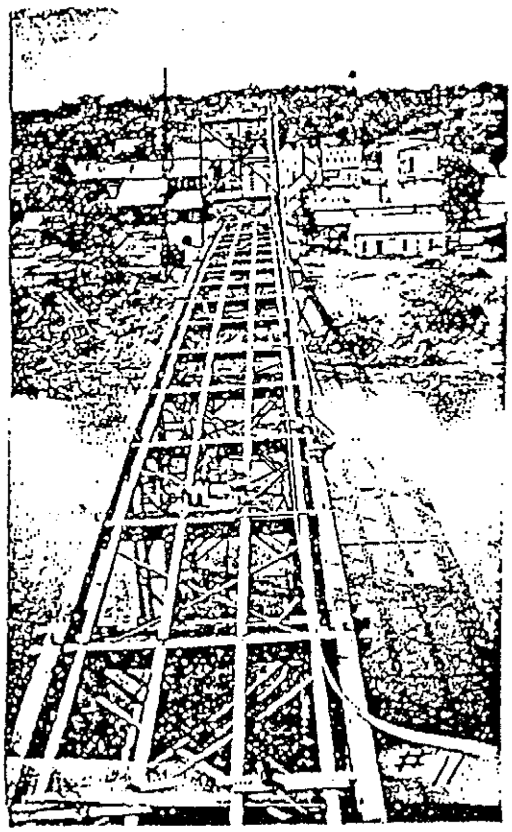
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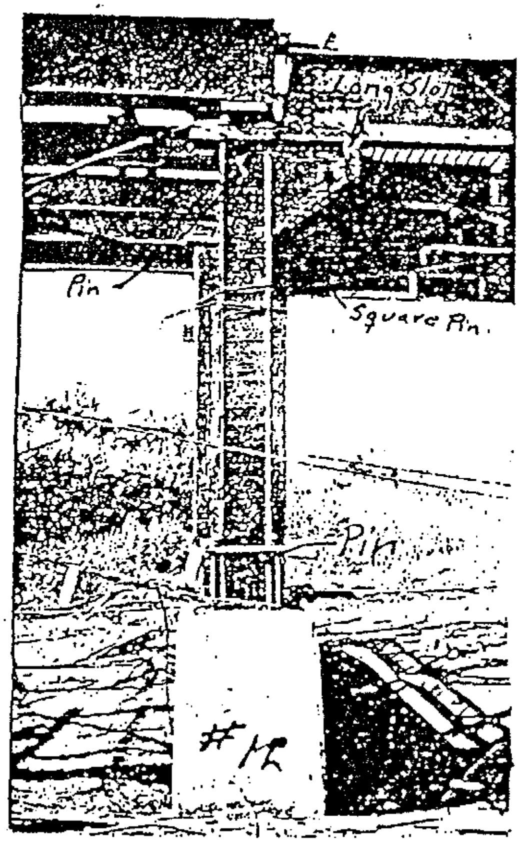
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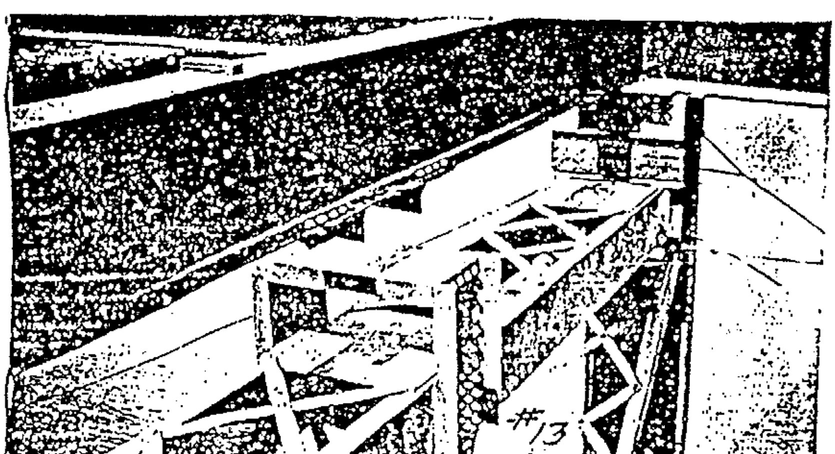
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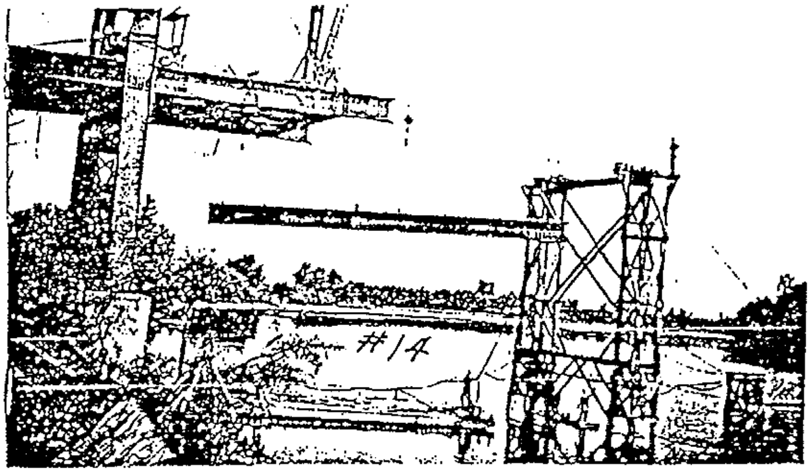
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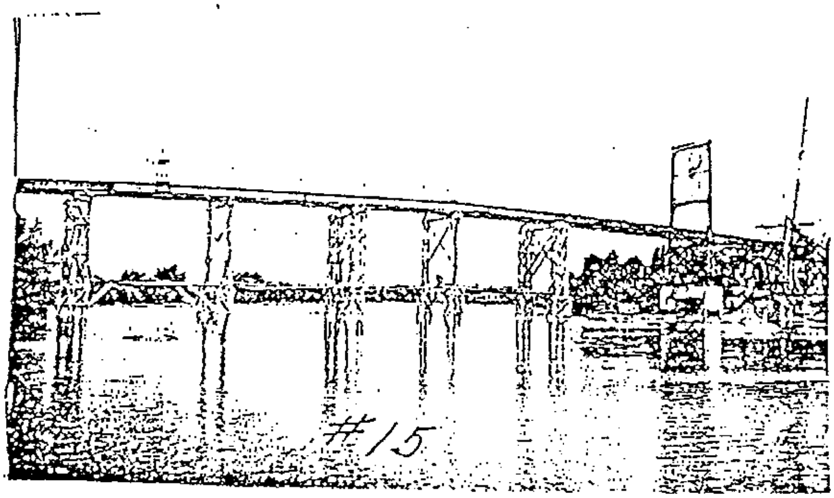
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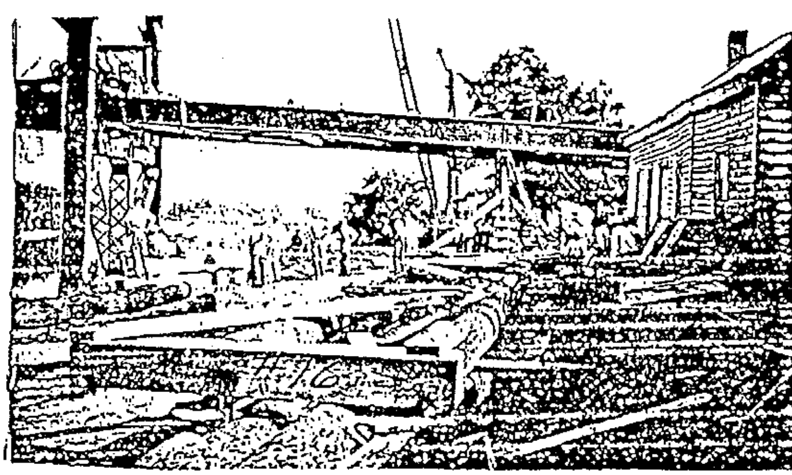
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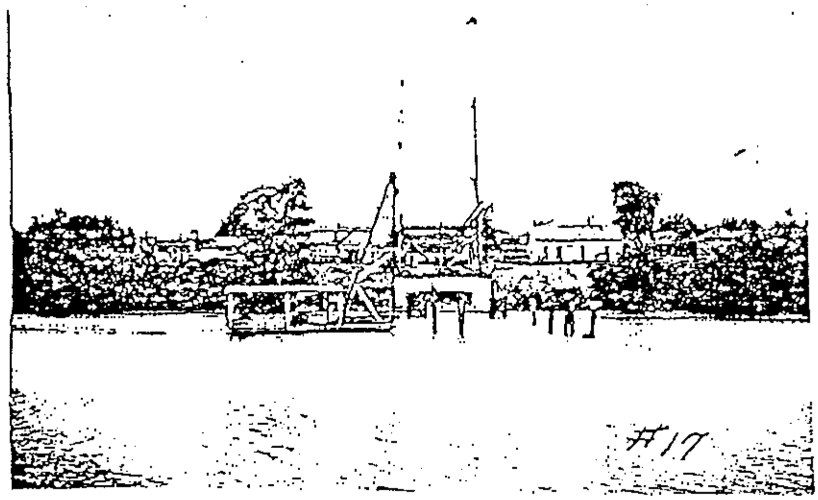
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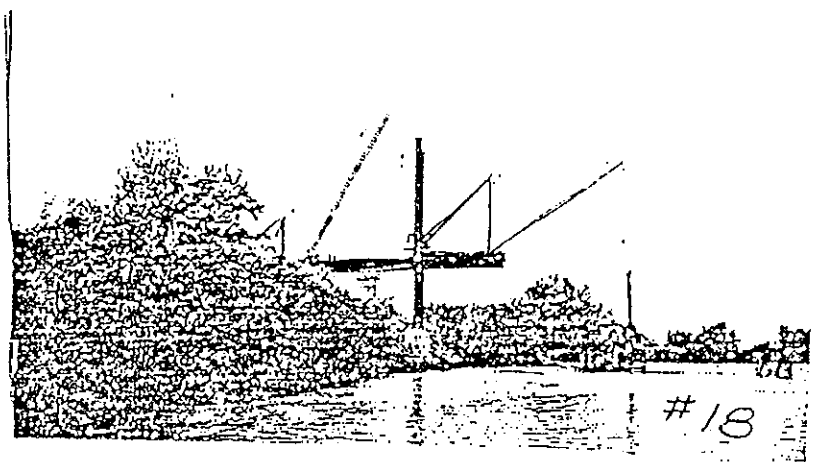
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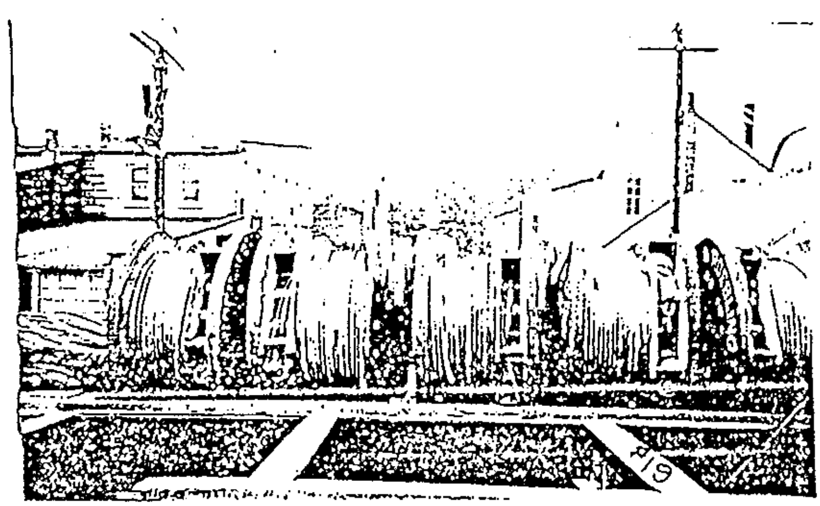
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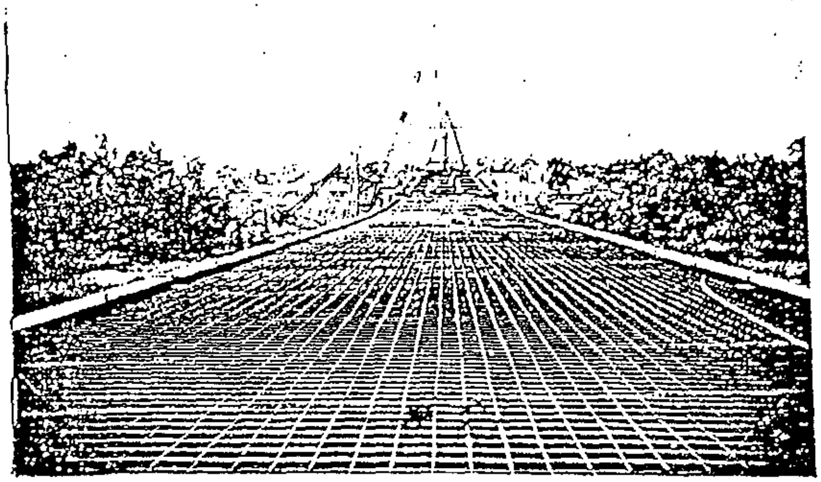


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